4 REMEDIATION DESIGN

This section describes the development of the Project's remedial design elements. As described in the previous section, the cleanup remedy involves placement of a stable engineered cap over localized areas of the landfill and the banks of Whatcom Creek. In addition, a wedge of stabilizing rock and gravel fill will be placed along the southern bank of the creek to mitigate against slope instability and refuse exposure during a design-level seismic event.

4.1 Refuse Excavation and Disposal

As generally described in the RI/FS and CAP, and consistent with the Comprehensive Strategy (Ecology 2000), refuse within a nominal 0.3-acre area within the existing B Street right-of-way (ROW) will be removed as part of the integrated cleanup and habitat restoration project, and the excavation area backfilled with a clean cap graded to relatively flat slopes. This will result in a net conversion of uplands into aquatic habitat, providing a substantial net gain in habitat area and function.

As part of this Project, fill and refuse material will be removed (likely using an upland excavator) and transported to and disposed at a permitted landfill (e.g., Roosevelt Regional Landfill) or whenever possible, recycled. Most of the excavation is targeted along the north bank of Whatcom Creek. Localized excavation will also be required for some areas along the south bank. Based on a review of soil and solid waste boring logs of the Holly Street Landfill Site (Appendix A), there are likely to be significant variations in density within the landfill debris; voids may also be present. During excavation some of the softer spots may slough when exposed. However, such behavior is expected to occur in isolated areas (not on a widespread basis). Moreover, as discussed in Appendix D, research indicates that the strength of landfill refuse is largely a function of strain, or the amount of movement during failure (Gabr and Valero 1995). Increasing strain leads to higher strengths –- a counterintuitive phenomenon that reflects the tendency of larger debris particles to interlock with one another during movement. Thus, the effect of sloughing is anticipated to be mitigated by the fact that the waste strength tends to increase with movement.

Careful controls will be implemented during construction as described in the accompanying Construction Quality Assurance Project Plan (CQAP) to ensure control of waste releases during the remedial action. The project specifications require that excavation be restricted to periods when water levels are at least one foot below the elevation of work activity. The only potential exception to this requirement is for areas along the south bank where excavation is required at elevations below elevation +3 feet Mean Lower Low Water (MLLW). In these cases, the Contractor will be allowed to perform in-water excavation only if they can demonstrate to the City, the City's Engineer, and Ecology that doing otherwise is infeasible. Furthermore, if in-water excavation is done in these areas, it will be subject to water quality monitoring and to observation by the City and Ecology.

Overall stability of excavated slopes will be maintained by limiting the proximity of equipment storage and soil stockpiling from the edges of excavated side slopes. There are also limitations on how long excavated slopes can remain exposed before backfilling is required. Freshly excavated surfaces will need to be rolled smooth before the next tidal inundation to reduce potential for erosion.

4.2 Control of Shoreline Seepage

The groundwater flow system at the Holly Street Landfill Site consists of a shallow unconfined aquifer within the refuse and underlying Recent Alluvial sediment (Anchor and Aspect 2003). Groundwater flow within this unconfined aquifer is generally directed from the upland areas toward Whatcom Creek. Fine-grained silts and clays present beneath the aquifer function as confining layers, restricting downward groundwater flow into deeper units.

Leachate within the refuse is generated from infiltration of incidental precipitation and from lateral inflow of groundwater into the landfill area. Tidal influence creates a sinusoidal groundwater flow path as the groundwater approaches the point of discharge into Whatcom Creek, and oscillates in response to tidally propagated waves. These groundwater oscillations are most pronounced within approximately 20 feet of the shoreline.

Monitoring conducted during the RI/FS, along with supplemental monitoring conducted as a part of the pre-remedial design evaluation (Appendix B) indicate that surface water cleanup levels set forth in the CAP for dissolved metals (copper and zinc) are currently exceeded in shoreline seeps along portions of the northwest lobe of the Holly Street Landfill. The geochemical data suggest that water within the Whatcom Creek estuary, high in dissolved oxygen, migrates into the shallow groundwater zone during high tides, creating oxidizing conditions within the saturated refuse. The oxidizing conditions promote mobilization of copper and zinc present within the refuse.

The Project includes removal of that portion of the refuse that encounters oxygenated water infiltrating from Whatcom Creek during high tides and placement of a sufficient thickness of semi-permeable shoreline cap. This design is intended to reduce concentrations of copper and zinc discharging to Whatcom Creek by displacing the zone of mixing outward from the refuse. Such displacement would separate the low dissolved oxygen environment within the refuse from oxidizing surface water, thereby reducing the release of dissolved copper and zinc.

For the purpose of Project design, a numerical groundwater flow model and integrated numerical groundwater contaminant transport model was developed to assess migration of dissolved oxygen (DO) inland from Whatcom Creek, considering advection, dispersion, and diffusion processes. Groundwater flow and transport model development and calibration are discussed in Appendix C; conservative model assumptions were incorporated to ensure the protectiveness of the remedy. The shoreline cap performance was evaluated by specifying a constant DO concentration boundary in cells representing Whatcom Creek. Two cap design scenarios were evaluated: 1) a medium sand cap with a uniform hydraulic conductivity of 0.02 cm/sec (the same as specified for the RI/FS); and 2) a less permeable silty sand cap with a uniformly lower hydraulic conductivity of 0.005 cm/sec. Multi-year transport simulations were performed with both configurations until a steady-state concentration profile was developed. These scenarios evaluated the relative effectiveness of the shoreline cap over a reasonable range of cap permeability values that may be specified in the design.

The cap performance scenarios indicated that a shoreline cap with a 5-foot effective thickness and a hydraulic conductivity of 0.02 cm/sec or less will greatly reduce oxygen flux from Whatcom Creek to adjacent shoreline solid waste deposits, relative to existing conditions. As shown in Figure C-5 (in Appendix C), the modeling analyses indicate that a medium sand (0.02 cm/sec) shoreline cap will result in at least a 95 percent reduction in DO concentrations encountering solid waste, as compared to existing conditions where no capping material is present. A less permeable silty sand, with a hydraulic conductivity of 0.005 cm/sec, would attenuate the influx of DO from Whatcom Creek by more than 99 percent, providing a substantially higher factor of safety for this design, with little impact on Project costs or constructability. Therefore, the less permeable material was selected as cap material for the project. With the shoreline cap in place, DO concentrations of groundwater in contact with the refuse will decrease substantially. Consequently, concentrations of zinc and copper in groundwater within the refuse will also decrease substantially, since both metals are less mobile at lower DO concentrations. As generally discussed in Appendix C, the predicted level of reduction in metals concentration is sufficient to achieve compliance with surface water standards set forth in the CAP.

4.3 Shoreline Cap Design and Construction

Consistent with the results of groundwater transport modeling described above, the total thickness of cap material to be placed during the Project must be 5 feet, measured in the general direction of groundwater flow. The flow direction is expected to be essentially horizontal. In order to provide an additional 50 percent factor of safety on cap protectiveness, the cap has been designed to provide an effective thickness of at least 7.5 feet in the groundwater flow direction. The desired 7.5-foot thickness of cap material in the horizontal direction of groundwater flow can be achieved by placing 2 to 2.5 feet of cap material on the proposed site grades, depending on the inclination of the capped grade. This geometric principle is illustrated in Figure 4-1.

The groundwater modeling demonstrated that 2 feet of cap material is more than sufficient for cap performance on relatively flat slopes of 4H:1V to 15H:1V (as will be present on the salt marsh bench area of the north bank). For steeper slopes of 3H:1V that will be constructed behind the rock berm on the north bank, a 2.5-foot-thick cap will be required.

The shoreline cap will be constructed in separate layers. The first layer will consist of 2 to 2.5 feet of clean, relatively fine-grained capping material, such as a slightly silty to silty fine sand or equivalent, which will have a permeability at or below approximately 0.005 cm/sec, as indicated by modeling results (Appendix C). The second layer will consist of a sand/gravel component of suitable grain size to resist erosive forces (see Section 4.5), and the final (surface) layer will consist of an imported topsoil.

In general, construction of the cap on the north bank of the creek will be limited to periods when water levels are at least one foot below the elevation of construction subgrade, and when there is no standing water present at the location of cap lift placement. Since the lowest elevation of cap material placement is +4 feet MLLW, the Contractor will need to sequence their operations with daily tidal fluctuations. An alternative approach will be allowed for placement of rock spalls and gravel at elevations below +3 feet MLLW. In these cases, the Contractor may elect to place the specified rock materials through the water, but subject to water quality monitoring by the City and Ecology. Furthermore, based on this monitoring, the Contractor may be required to employ Best Management Practices (BMPs) for turbidity control as well (i.e., silt fencing).

The Contractor will be required to achieve a nominal degree of compaction on each lift of cap material underlying the topsoil layer by rolling each lift with a roller or heavy construction equipment. The topsoil layer will receive only a light tamping, since this compaction could adversely affect its ability to support vegetation.

The upper bank area will be covered with a biodegradable coir erosion control fabric and planted with woody riparian vegetation (native trees and shrubs) since it is above the area of normal tidal inundation.

4.4 Softening and Stabilization of South Bank

As part of remedial measures for the Holly Street Landfill, a rock and gravel "buttress" will be placed along the south bank slopes of Whatcom Creek with a design grade of 2H:1V or flatter. This will serve to both "soften" the currently eroded escarpment geometry of this bank, and increase its overall stability, including increased stability against failure during seismic events. Where existing bulkheads are present, this will require a maximum of about 10 feet of rock and gravel material (measured vertically at the slope face), which will supplement the wooden piles in providing support for the slope. In some areas, excavation and off-site disposal of solid waste from the South Bank is included in the Project design, in part to maximize habitat-related benefits (see accompanying Project Plans).

As discussed in the previous section, rock and gravel placement will generally be sequenced to occur above water levels. However, the Contractor may elect to place the initial lifts of rock and gravel through the water, subject to specified monitoring requirements.

Stability analyses of the slope with the proposed buttress (Appendix D) indicate that the buttress will increase the slope factor of safety by about 40 to 50 percent. These analyses assume that the wooden piles have been left in place (cut off at the mudline) as part of the remedial measure. It is important that the existing piles not be pulled during construction for two reasons:

- The piles currently provide additional stability for the slope, particularly for potential seismic events and will continue to do so after the sand and gravel buttress is placed; and
- Pulling the piles would tend to cause additional unnecessary stress within the slope that could precipitate localized sloughing during removal.

Insert figure 4-1 here

4.5 Erosion Protection

Under current conditions, the site experiences erosion and periodic flooding due to stream flows and tidal influence. Design of the reconfigured site needs to account for such forces by incorporating armor materials that can resist anticipated erosive forces. This section presents the basis for selection of suitable armoring materials for the cap and reconfigured banks.

4.5.1 Evaluation of Erosive Forces at the Site

The site is located at the mouth of Whatcom Creek, upstream of the Whatcom Waterway (Figure 2-1). Typically, the types of potential erosive forces that are evaluated to ensure long-term cap stability in aquatic environments include stream flows, tidal flows, and wind or vessel-generated waves. However, because of the sheltered and non-navigable setting of the site, wind and vessel-generated waves are not significant, and are therefore unlikely to influence cap stability. Furthermore, wind waves coming in from Bellingham Bay cannot reach the location of this site because of the constriction at the Holly Street bridge and the relatively shallow depths of this area. Therefore, stream and tidal flows have been identified as the main factors contributing to potential erosive forces at the site.

4.5.2 Calculation Procedure

The required particle size gradation for cap and surface protection was determined using velocities computed within the creek channel for three different tide levels: mean low water level, mean tide level, mean higher high water level. These velocities were increased by a factor of 50 percent to provide an additional factor of safety (consistent with cap design methodology) to allow for higher velocities on the outside bends (U.S. Army Corps of Engineers 1991).

This design-level erosion analysis is expected to be additionally conservative because it does not expressly account for the fact that shallower side slopes tend to result in dissipation of velocity through turbulence, eddy formation, and friction. Above an elevation of +6 feet MLLW, the north bank at the site will generally be constructed at a shallower angle than below this elevation. The change of the slope creates a bench with a relatively shallow water depth, which will experience slower velocities, since most of the velocity will occur in the deeper portion of the main channel. Thus, this conservative analysis provides an additional factor of safety in the design.

For each design velocity, four different methods and diagrams were used to compute stable sediment size: Plate B-28 from the Engineering Manual EM 1110-2-1601 (later referenced as *Method 1*), the Hjulström (1935) and Shields (1936) diagrams (*Method 2* and *Method 3*), and Figure 5-5 of the Engineering Manual EM 1110-2-1418 (*Method 4*). All these diagrams are presented in Appendix E. For the Shields diagram, a dimensionless shear stress of 0.03 was used, and bottom shear stress was computed using the following formula:

$$\tau_b = \frac{1}{2}\rho f V^2$$

where ρ represents water density

f is a friction factor, equal to 0.03

V represents the design velocity

The results obtained with the different analyses were then compared to determine a stable rock size for each elevation range.

4.5.3 Flow Data and Calculations

Whatcom Creek inflow at the site is a combination of water originating in Lake Whatcom, tributary creeks, the adjacent fish hatchery, stormwater, and from tidal flows originating in the Whatcom Waterway. Currently, no gage has been installed at the site to record flow data. The peak stream flow measured at U.S. Geological Service (USGS) gage 12203500 on Whatcom Creek (upstream of the site) was 1,350 cubic feet per second (cfs) in 1950. The flow used by the Federal Emergency Management Agency (FEMA) for a 100 year flood condition on Whatcom Creek is 1,429 cfs (FEMA, 1982). This 100-year flow rate was used for the purpose of cap design.

Tidal flow contributions were also considered in the computation of potential flows and resultant bed velocities within the Whatcom Creek channel. The maximum flow velocity was computed for different water levels, since the river cross-sectional area changes with water surface elevation at different tidal stages, with a corresponding effect on bed velocity. The post-construction grading plan was used to determine cross-sectional areas.

Because tidal flows vary substantially over the tidal cycle, peak ebb and flood tide currents were calculated to correspond with maximum tidal exchange period. The maximum tidal flow velocity at ebb tide was found to be approximately 0.1 feet per second, which was added to the peak measured flow velocity in Whatcom Creek to determine the design velocity. Clearly, at this site, flood flows were determined to be more significant than tidal flows as contributors to erosional force.

4.5.4 Armor Requirements

The cap armor analyses were performed using the methods described above for three different tidal conditions and their corresponding water levels and velocities. The calculated water velocities at three tide levels in the design flood event are presented in Table 4-1. The four different methods led to different sediment sizes, as presented in Table 4-2.

Table 4-1
Design Velocities in Whatcom Creek

Peak River Flow (feet^3/second)[a]	Tide Level (feet)[b]	Average Cross-Secondtional Area (feet^2)[c]	V_{River} (feet/second)[d]	V_{Tide} (feet/second)[e]	V_{Total} (feet/second)[f]	V_{Design} (feet/second)[g]
1,429	MHHW: 8.5	854	1.67	0.14	1.81	2.7
1,429	MTL: 5.1	464	3.07	0.08	3.15	4.7
1,429	MLW: 2.5	221	6.44	0.04	6.48	9.7

Table 4-2
Stable Sediment Size and Type in Whatcom Creek

Tide Level (feet)[b]	V_{Design} (feet/second)[g]	Method 1 D_{50} (inches)	Method 2 D_{50} (inches)	Method 3 D_{50} (inches)	Method 4 D_{50} (inches)	Design D_{50} (inches)	Elev. Range (feet)
MHHW	854	0.6	0.4	0.2	—	0.6	8.5 and above
MTL	464	1.6	0.7	0.6	0.4	1.6	5.1 to 8.5
MLW	221	7.0	3.2	2.7	3.2	7.0	Bed to 5.1

a Peak River Flow is 100 year flood event as defined by FEMA.
b Tide level is shown for Mean Higher High Water (MHHW), Mean Tide Level (MTL), and Mean Lower Low Water (MLLW) conditions.
c This column gives the average post-construction cross-sectional area of Whatcom Creek in the site area for the three different tide elevations.
d V_{River} is the velocity of the water at the three different tide stages. This velocity was computed using the Peak Flow and the cross-sectional area, for different tide elevations.
e V_{Tide} is the ebb tide water velocity for the different tide elevations.
f V_{Total} is the sum of V_{River} and V_{Tide}.
g V_{Design} was computed by multiplying V_{Total} by 1.5

The erosion analyses indicate that the banks may need to be protected with large cobble or spalls at and below approximately +5.1 feet MLLW in order to ensure their stability during the 100-year flood condition. Above this elevation, the required armor size becomes smaller with increasing elevation, with a coarse gravel required at an elevation at and below approximately +8.5 feet MLLW. At upper intertidal elevations (+8.5 feet MLLW and above), a fine gravel was determined to be stable.

Potential erosive forces were further addressed in this design by specifying construction of a rock berm along the north bank, which will protect the adjacent north bank shoreline from both tidal and flood-induced peak flows. The lower elevations of the rock berm (at and below roughly +5 feet MLLW) require armoring with a spall-sized material.

4.5.5 Incorporation of Armor Into Cap Design

The rock armoring described above has been incorporated into the constructed caps on the north and south banks. On the north bank, placing the required rock armor directly on the cap surface would conflict with the goal of establishing vegetation. Therefore the armoring material will be placed below the topsoil and cap layers, in a (minimum) 6-inch-thick layer between the cap material and the surface topsoil layer. This buried layer of armoring material will act as a protective barrier against erosion of the cap in the event of a design-level flood event, thus cap preventing erosion and potential exposure of refuse should the flood erode the overlying topsoil layers. It is expected that the surficial topsoil layer will remain stable during most conditions, particularly after the stand of vegetation has been established.

Additional protection against erosion of the north bank will be provided by the following design features:

- Construction of a rock berm along that portion of the bank that encounters the highest flows
- Establishment of a stable stand of vegetation in the surfacial cap topsoil
- Placement of a biodegradable erosion control fabric (coir) on the surface of slopes inclined at 4H:1V or steeper

On the south bank, the constructed rock buttress will be composed of a rock size sufficient to resist erosion, as described above. The surface of the rock buttress will be covered by a layer of gravel that will be more amenable to safe public access. If a design-level flood event removes some of this gravel layer, then the remaining armoring rock will remain to resist further erosion of the south bank.

4.6 Upland Cap Design

The potential for human and environmental exposure to refuse and associated soil contaminants will be controlled through construction and maintenance of a minimum 2-foot-thick permeable cap or equivalent direct contact exposure barrier. A soil cap meeting this specification is already in place throughout the southeast lobe of the landfill (Maritime Heritage Park) and in most of the northwest lobe of the site.

However, based on pre-remedial design sampling data, in limited areas of the site, the existing cap is insufficient (i.e., less than 2-feet-thick and also not overlain by asphalt or concrete barriers), and requires augmenting to meet containment specifications set forth in the CAP. Localized areas within the Maritime Heritage Center (fish hatchery) contain only a thin cover (less than 2 feet thick) and therefore will require a cap amendment. The delineated capping area is depicted on the accompanying Project plans. The upland cap will be constructed concurrent with the shoreline remedy.

Below elevation +10 feet MLLW, the upland cap area will be first excavated and then capped so that the minimum 2-foot-thick cap thickness is achieved without modifying currently existing grades, thereby incurring no net loss of aquatic area. The cap will be carried down to elevation +6 feet MLLW. Above elevation +10 feet MLLW, minor regrading will be accomplished to provide trail continuity.

4.7 Water Quality Protection

Water quality controls will be implemented as a part of this action. Dredge elutriate testing conducted on composited sediment from the site (see Appendix B) indicated that the only possible exceedances of screening levels would be from the particulates generated by turbidity releases. Therefore if turbidity is controlled, water quality standards will be met. Turbidity releases will be prevented by restricting in-water work windows to low tide conditions, and using erosion control BMPs such as rolling and smoothing freshly excavated surfaces. These controls are described in more detail in the accompanying specifications and CQAP.